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COMBINED STRUCTURE SPIDER NEST AND REINFORCED CONCRETE PILE AS AN ALTERNATIVE STORY BUILDING FOUNDATION

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ABSTRACT

The combined of Spider Nest Foundation (KSLL) with Reinforced Concrete Pile, is a combination of shallow foundation with deep foundation, is one of the alternative modification of foundation for middle level building with maximum floor number 8 levels. This design modeling foundation rib KSLL as the bearing wall with a cantilever-flops on the floor plate. Referring to the principle of mechanics with the clasp on the Plate, the worst condition when the other side of the rib has not been filled so that the rib will accept one side of the soil pressure, will result in the cantilever flexure moment as the base for reinforcement. With the addition a number of pile diameter of 50 cm at the corners of the building to minimize the decline. Based on analysis of the carrying KSLL capacity with safety factor SF=4 obtained q_{alow} *is 1426* kN/m^2*, and* q_{ult} = *5703,2* kN/m^2*, after being given a pile strengthening, the carrying capacity increased to 14092 kN / m2. The result of determining KSLL declining analysis is a long-term consolidation decrease of 14.4 cm qualify < 15 cm. In the combined system of KSLL Foundation with reinforced concrete piling pole as deep as 11.00 m, the value of decrease is only 7.92 cm, so that the structure of the foundation of KSLL combined with Piling is very stable and qualified as the foundation of 8-storey building.*

Keywords: Combination KSLL, Piling, Foundation, Bearing Capacity, Settlement.

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1. INTRODUCTION

Soil improvement is required if the original soil bearing capacity is poor so that the top soil layer needs to be replaced with soil that has a better carrying capacity (Hidayatun et all, 2018). One solution is the foundation of spider nest Construction (KSLL) is a structure that utilizes soil media as part of the foundation structure which is able to anticipate the seismic forces both horizontally and vertically (Raharja and Sutjipto, 1984; Raharja, 2015). Called the Spiders Nest because the reinforcement of the foundation plate around a column is shaped interlocking webs monolithically resembles a spider Nest (Figure 1).The foundation plate is given a monolithic diagonal ribs support renderer so that the structure becomes a rigid that serves as a burden of gravity and earthquake loads. The foundation structure is relatively lighter because some of the volume of concrete is replaced by compacted soil.

Figure 1 The Idea of KSLL Structural Foundation (Education Media, 2018)

The KSLL foundation will uniformly decrease because the construction consists of a thin plate with a reinforced concrete ribs actuator that serves as a stress spreader due to the column load of the upper structure. Because KSLL is a shallow foundation category that carries the burden of Building structures up to 8 levels, a high enough settlement can not be avoided, although it does not lead to significant disruption to the stability of the building structure. So in this paper arises the idea to combine KSLL structure with reinforced concrete piles. We all know that the pile foundation when it comes to the depth of the stable point of the soil, the decline is very small. To be more economical (Samudro and Mangkoedihardjo, 2006; Sedayu and Mangkoedihardjo, 2018), the pile configuration will be installed at the exterior column points around the edge of the KSLL.

2. METODOLOGY

Building Modeling Design and dimensions are as follows:

Building Data

1. Number of Floors : 8 lantai + 1 basement	6) Wide of Foundation KSLL : 34,55 m
2. Deep of Basement : 2,3 m	7) Tall of Building: 30,04 m
3. Length 0f Building : 40 m	8) Typical Floor height: 3,20 m
4. Length of Foundation KSSL: 44 m	9) Main Structure: Reinforcement Concrete
5. Wide of Building : 32,55 m	10) Sub Structure: Combined KSLL and Pile

The Modeling Structure result

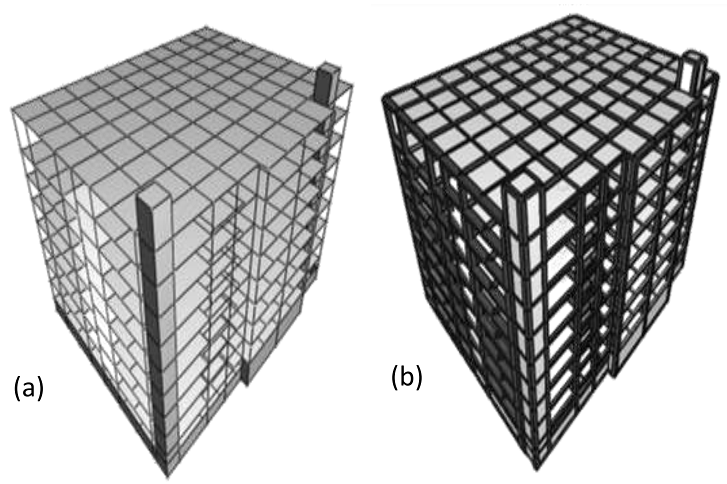
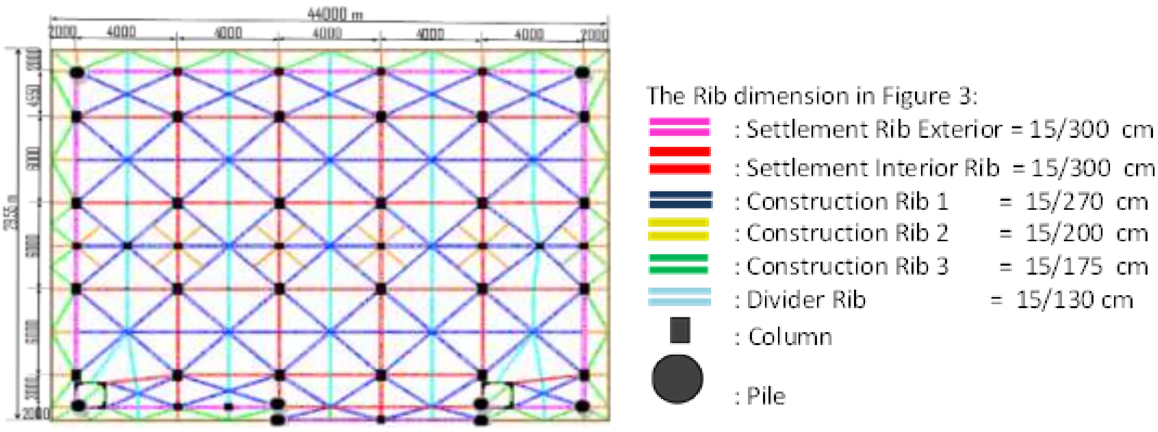


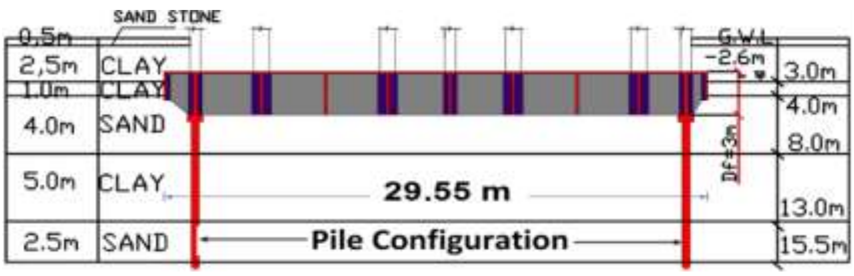
Figure 2 Structural Modeling of Building 8 floors

Foundation Design

The result of combination design of KSLL Foundation with Pile is like Figure 3 below:



Top View



Cross Section

Figure 3 Foundation Design

4. RESULTS AND DISCUSSION

Large loads that work on Structure

Total load with a combination of loading:1 DL+1 LL, The combination of the largest loading components is used as the ultimate load to be used in foundation design

Table 1 The Loads Combination, 1 DL+ 1 LL

Column	Loads P (kN)	Column	Loads P (kN)
P1	1691	P28	7405
P2	2828	P29	6898
P3	2838	P30	6915
P4	2836	P31	7347
P5	2822	P32	3482
P6	1678	P33	3145
P7	3752	P34	3925
P8	6384	P35	6620
P9	6484	P36	5958
P10	6483	P37	5259
P11	6387	P38	2132
P12	3772	P39	956
P13	3597	P40	1927
P14	7627	P41	725
P15	6886	P42	1125
P16	6885	P43	1348
P17	7504	P44	1051
P18	3511	P45	1125
P19	456	P46	1299
P20	28	P47	2472
P21	2635	P48	1236
P22	3928	P49	966
P23	3927	P50	1685
P24	2670	P51	2090
P25	118	P52	1186
P26	866	P53	914
P27	3537		

Table 2 Effective Soil Pressure

Depth (m)	γ'(kN/m^3)	Po(KN/m^2)	Depth(m)	γ'(kN/m3)	Po(KN/m2)
0	Basement (-2,3 m)		16	9,220118	124,4771
1			17	10,09167	134,5688
2			18	10,09167	144,6604
3	The soil Fillers embankment RIB		19	9,076136	153,7366
4			20	12,05037	165,7869
5	11,97778	11,97778	21	12,05037	177,8373
6	11,97778	23,95556	22	11,69	189,5273
7	12,05037	36,00593	23	11,69	201,2173
8	12,05037	48,0563	24	11,69	212,9073
9	12,05037	60,10667	25	11,52535	224,4327
10	9,220118	69,32679	26	8,798925	233,2316
11	9,220118	78,5469	27	8,798925	242,0305

12	9,220118	87,76702	28	8,798925	250,8294
13	8,242051	96,00907	29	8,798925	259,6284
14	9,623952	105,633	30	8,798925	268,4273
15	9,623952	115,257			

Effective Soil Pressure

Effective Soil pressure, tabulated as follows

The calculation of effective soil stress (Po) at a depth of 30 m is 268.43 kN / m2 (268,43 kPa), like tehe Table 2.

The Maximum Soil Stress

The length of the foundation plate KSLL (L) = 44m, Wide (B)=29,55m, Thick plate(D) = 0,15 m, Depth rib foundation = 3.0 m from the bottom of the basement floor.

Calculate the Stress then the qo_{max} is 127,76 kPa

KSLL Bearing Capacity Analysis

Based on soil data, KSLL foundation rests on a 5,0 m thick sand soil, with cohesion C = 0 and sand shear force $\phi=28,578^{o}$, then Nq=15,689, Nc=26,97, and Nγ=12,33. Shape factors, depth, and slope according to Mayerhof are: Kp=2,8341, sc=1,445, sq= 1,22, sγ=1,22 and dc=1,03, dq=1,01, dγ=1,01and are q=33,562 kN/m^{2,} so q_{ult}=5703,2 kN/m^2, and qa_{KSLL}=1425,75 kN/m^2, Safety Factor: SF=16,24

Stress Controls Occurred

Building load divided by the area of the foundation, to calculate the magnitude of the voltage. Calculate the stress at the bottom of the foundation based on uniform load:

$$q_{o_{equally}} = \frac{\Sigma P}{A} = \frac{181320{,}56\ kN}{44 \times 34{,}55} = 119{,}3\ \text{kPa}$$

Settlement Analysis of KSLL

Result of instantaneous Sttlement (Si) on sand soil is: 253. 10^{-4} m = 25,3 mm (see Figure 4).

Settlement on soil Clay

The result: Si_1 = 34 mm and Si_2 = 15 mm

The results of calculation of total Settlement immediately on clay is: Si = 34 – 15 = 19 mm

Then, immediately Settlement = Si sand soil + Si Clay = 25,3 mm + 19 mm = 44,3 mm (see Figure 5).

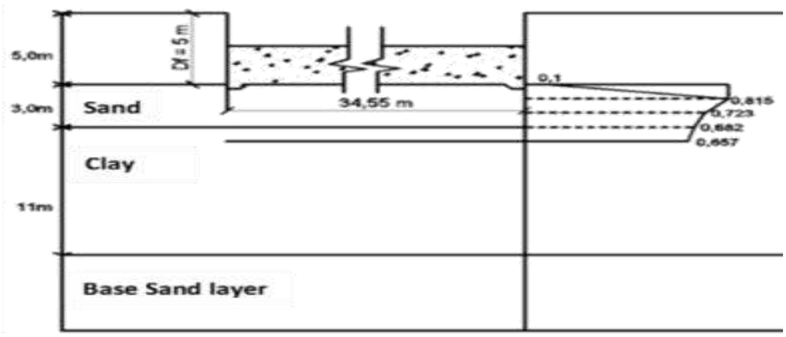


Figure 4 Settlement Sketch on Sand soil

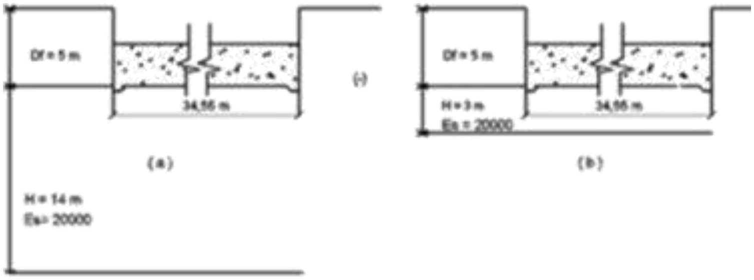


Figure 5 Settlement Sketch on Soil Clay

Settlement in Primary Consolidation

The amount of primary consolidation settlement calculated from a depth of 9.0m is provided in Table 3.

Table 3 Consolidation settlement

Depth(m)	e	Cc	P0	∆P	scp(m)	scp(mm)
9	0,78	0,182	60,107	29,384	0,01770	17,704
10	0,69	0,162	69,327	28,617	0,01436	14,356
11	0,69	0,162	78,547	28,617	0,01291	12,907
12	0,95	0,223	87,767	27,851	0,01366	13,662
13	0,68	0,159	96,009	26,701	0,01011	10,106
14	0,68	0,159	105,633	26,190	0,00912	9,122
15	0,69	0,162	115,257	25,296	0,00824	8,243
16	0,68	0,159	124,477	24,146	0,00730	7,302
17	0,76	0,178	134,569	23,635	0,00711	7,110
18	0,76	0,178	144,660	22,740	0,00642	6,415
19	0,55	0,128	153,737	22,229	0,00487	4,867
20	0,55	0,128	165,787	21,080	0,00431	4,313
21	0,40	0,094	177,837	20,441	0,00316	3,163
22	0,40	0,094	189,527	19,036	0,00278	2,782
23	0,40	0,094	201,217	18,397	0,00254	2,543
24	0,42	0,098	212,907	18,397	0,00249	2,494
25	0,86	0,201	224,433	15,969	0,00323	3,234
26	0,86	0,201	233,232	15,969	0,00312	3,116
27	0,86	0,201	242,031	15,969	0,00301	3,006
28	0,86	0,201	250,829	14,947	0,00272	2,723
29	0,86	0,201	259,628	13,159	0,00233	2,326
30	0,86	0,201	268,427	13,159	0,00225	2,252
Scp =					0,14375	143,749

Then the large decrease in total is the amount of instantaneous decrease in sand, clay and consolidation:

St=Si+Scp=44.3mm+144mm=188mm.

Long-term decrease is limited to a maximum of 15 cm (Mina et al, 2017). The results of the analysis of the instantaneous and long-term decrease of > 15 cm is 18.8 cm, but still meet the criteria due to soil compaction inside the ribs, so the soil under the foundation also solid. Thus the value of the instantaneous decrease can be ignored, so the most decisive decline is the decrease in long-term consolidation = 14.4 cm (see Figure 6).

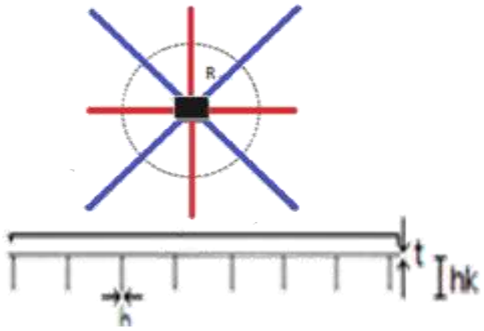


Figure 6 Relationship of Rib KSLL with Column P14

Calculation of the maximum load on Rib

Column = a_1 x a_2 = (100 x 70) cm^2; Slab thick, t = 15 cm; Rib thickness, b_1= 18 cm (the red color)

hs = 300 cm; Rib thickness, b_2 = 18 cm (the blue color); hk = 270 cm; $\pi R^2 = {}^{P}/{qa_{KSLL}}$

$$R = \sqrt{\frac{7526}{\pi \text{ x } 1426}} = 1{,}3 \text{ m} = 130{,}5 \text{ cm} \approx 140 \text{ cm}$$

$$y = \frac{\begin{bmatrix} t(2\pi R).\frac{1}{2}t + 4b_1.(hk-t).\left(\frac{hk-t}{2}+t\right) \\ +4b_2.(hs-t).\left(\frac{hs-t}{2}+t\right) \end{bmatrix}}{\left(2\pi R.t + 4b_1.(hk-t) + 4b_2.(hs-t)\right)} = \frac{\begin{bmatrix} 15(2\pi.140).\frac{1}{2}15 + 4.18.(270-t).\left(\frac{270-15}{2}+15\right) \\ +4.18.(300-15).\left(\frac{300-15}{2}+15\right) \end{bmatrix}}{\left(2\pi.140.15 + 418.(270-15) + 4.18.(300-15)\right)}$$

$$y = \frac{98910 + 2616300 + 3231900}{13188+18360+20520} = \frac{5947110}{52068} = 114{,}28 \text{ cm}$$

Ix =441951270,4 cm^4, and t_e is 182 cm (rounded)

Thickness the Rib = 0,41 m. Load maximum on Rib obtained 18547,486 kN, and P_1 = 15101,211 kN. So, the total load on Rib P=Pmax – P_1=18547,486 –15101,211 = 3446,28 kN, then the Stress in Rib is: $q = \frac{Pmax_{Column}}{qa_{KSLL}}$. Area F calculated = 13 m^2

P_{max} of Column=7627kN on Column P_{14} (see Table 1), check:$q = \frac{P_{working}}{F} \leq q_{izin}$;$\frac{7627 \text{ kN}}{13 \text{ m}^2} \leq 1426 \text{ kN/m}^2$,

the result is: 586,325 kN/m^2 $\leq$ 1426 kN/m^2 … OK!

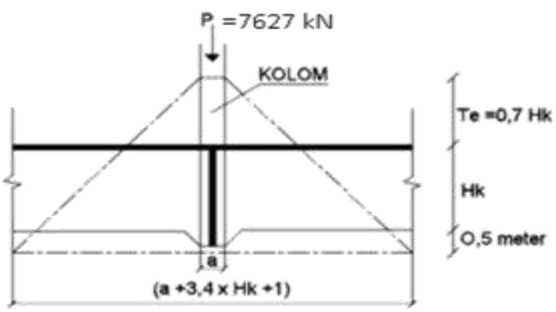


Figure 7 Area of column load P14 distribution before any Moment load

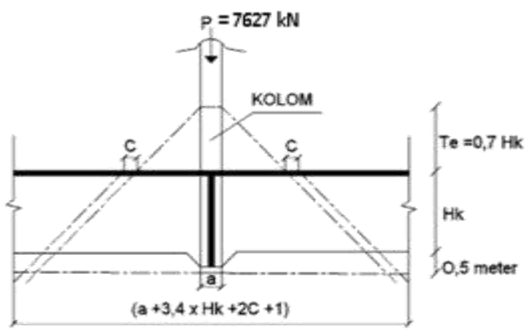
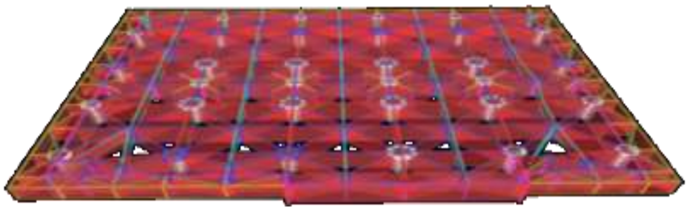


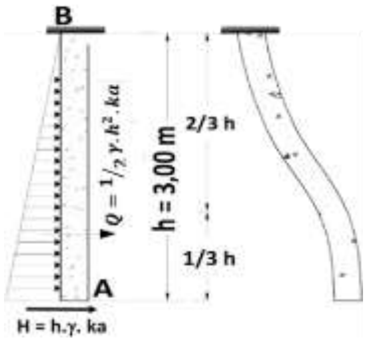
Figure 8 Area of column load P14 distribution after any Moment load

Rib and Plate Modeling



In the reinforcement needs analysis, calculate the loads and reactions that occur in the rib manually. Where the rib is assumed to be a cantilevered wall, with one free end and the other end wedged on the plate, the rib wedged on the plate will remain perpendicular to the floor slab. Rib resist lateral loads due to soil pressure, the maximum moment will occur on the pedestal pinch. Install the main reinforcement in the vertical direction rib, and the mounting reinforcing is horizontal direction.

Rib Reinforcement



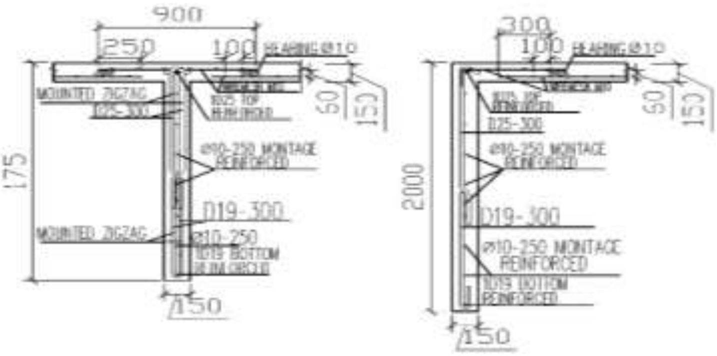
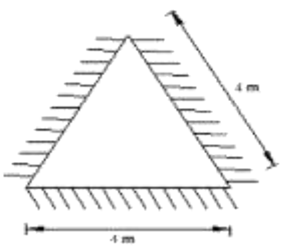
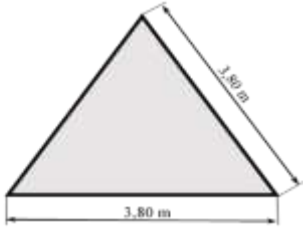


Figure 10 Load and Deformation diagram of Rib **Figure 11** Reinforcement Rib Interior and Exterior

Slab Reinforcement

Triangle shape of Slab





Support area Field Area

Figure 12 Support Condition **Figure 13** Field Condition

On Support Area: Wu = 19,224 kN/m (has included its own load); Mu $=$Wu $.L^2/86,85 = 19,224.10^{6}\cdot 4^2/86,85 = 3,54.10^6$ Nmm. With the addition 14%, then Mu = $4,04.10^6$ Nmm, f'c = 25 Mpa; and fy = 350 MPa

d'(Concrete covers)= 40 mm. Used wiremesh, M10 with section area: A_S=471,24mm^2.

On Field Area: Mu $=$Wu $.L^2/50,85=19,224.10^6.4^2/50,85=5,46.10^6$ Nmm; With the addition 14%, then Mu = $6,22.10^6$ Nmm. Used wiremesh, M10 with section area: A_S=471,24mm^2

Reinforcement Slab an around column

f'c = 25 Mpa; and fy = 320 MPa; d'(Concrete covers) = 40 mm

Using yield line theory, the ultimate moment is obtained, due to maximum central load:

Mu= $\frac{P}{2.n\tan\left(\frac{\pi}{n}\right)} = \frac{7627,437}{2\ x\ 8\ x\ \tan\left(\frac{\pi}{8}\right)}$ = 1150,89 kNm.

The load P is considered to work on limited area acting on an area bounded by radius = r, then it is obtained:

M' + M = $\frac{\mathrm{P}}{2.\pi}\left(1-\frac{12.p}{3.r}\right)$; M' + 1150,89 kNm = $\frac{7627,437}{2.\pi}\left(1-\frac{12.1,4}{3.1,295}\right)$

M' = -4035,53 / 1150,89 kNm = - 3,506 kNm

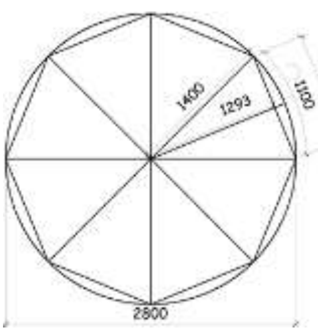


Figure 14 Radius Reinforcement Slab

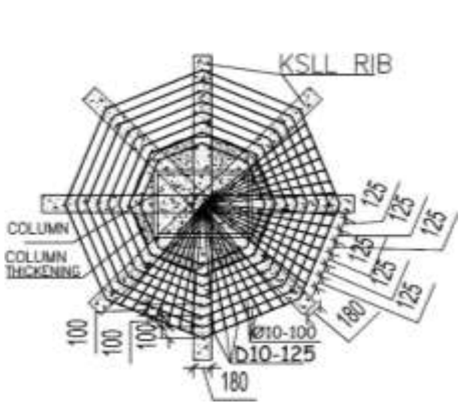


Figure 15 Reinforcement Slab an aruon Column

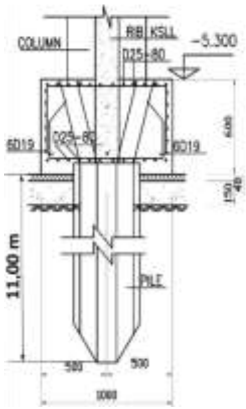


Figure 16 Detail of Pile

With the addition 14%, then Mu = 4.10^6 Nmm, used ∅10-125 longitudinal direction, A_S= 550 mm^2.

Ø8-100 cross direction, with A_S= 502,65 mm^2.

Calculation of Single Piling Capacity

The upper structural load is borne by the KSLL foundation, so the load on the pile is the foundation load of KSLL+workload obtained from the Structural modeling.

The upper structural load is borne by the KSLL foundation, so the load on the pile is the foundation load of KSLL+workload obtained from the Structural modeling.

The pile to be used is a "spun" pile as follows:

Outside diameter (DL) = 50 cm - Length of one pile segment = 6 m

Inner diameter (DD) =32 cm - Concrete thickness = 9,0 cm

Concrete Compressive Strength (f'c) = 60 Mpa - Steel yield Stress (fy) = 400 MPa

Nb end bearing pile = 20; Ab = 1/4 x 3,14 x 0,5^2 = 0,196 m^2; Nav = $\frac{16+2+2+3+4+5+8+14+20}{9}$ = 6,75

As = 3,14 x 0,5 x 11 = 17,27 m^2; Pu = 40 x Nb xAb+ $0.2N\ ave.As$

Pu= (40 x 20 x 0,196)+0,2 x 6,75x17,27=2264,7kN; Pu < 380xNavxAb=380 x 6,75 x 0,196 m^2 =5027…OK

Calculated Q_{ult} = 1143,5 kN. Carrying capacity 1 pile =1143,5 kN, to minimize the decline, at the corners of the building (at 8 point columns) required 1 pile (see Figure 16).

Decrease KSLL Combination Pile

Stiffness component Kr=11334,7 kN/m, and stiffness KSLL: Kp=1918 kN/m

The foundation structure of KSLL is always synergic with each other, with the soil compaction inside the ribs then the empty space will be filled so that the pile planting in the corners of the building is assumed like a table, each point requires 1 pile, but all are interconnected with each other to form a group of piles. Counting stiffness pile group: $Kp_{sys}=Kp\sqrt{np}=1918\sqrt{8}=5424$kN/m, stiffness pile-raft: Kpr=4963,83 kN/m

Proportion of load borne by the raft-pile: a = 0,03914, then βp = 0,96, All points use each 1 pile then the magnitude of P_1 = 1194 kN. Because of the burden at all points of the pile: P < P1 then decrease at each point of the pile using the equation: $S = \frac{P}{Kpr}$, in the table as follows.

Table 4 Decrease KSLL Combination Pile

Pile point	P (kN)	S (m)	S(cm)	Pile point	P (kN)	S (m)	S (cm)
1	389	0,0784	7,84	5	168	0,0338	3,38
2	393	0,0792	7,92	6	170	0,0342	3,42
3	135	0,0272	2,72	7	150	0,0302	3,02
4	190	0,0383	3,83	8	265	0,0534	5,34

So the highest decrease of 7.92 cm, for the combined carrying capacity by both foundations of: $Q_T = Qa_{ksll} + \sum((Q_p+Q_s)\ /(0{,}25.\pi.0{,}5^2.11)) = 5703{,}2\ \mathrm{kN/m^2} + (\ 8.(\ (1099 + 1165{,}7)\ /2{,}16) = 5703{,}2 + (8.1048{,}55) = 14092\ \mathrm{kN/m^2}$

The Pile-cap Reinforcement

Pile-cap dimensions: 1x1x0,5m,using the main reinforcement D25-125 and montage reinforcement D19-250.

CONCLUSION

1. The upper Building load is supplied to the foundation through column points, the total load of 181320.56 kN, and Pmax is in column P14. amounted to 7627.437 kN
2. The carrying capacity of the KSLL foundation on safety SF = 4 to resist settlement, yields qaKSLL = 1426KN / m2 with qult = 5703.28 kN / m2, after the addition of the pole to 14092 kN / m2.
3. Soft clay layer under sand layer still decrease consolidation.
4. The resulting decline in total (immediate+long term) on the foundation KSLL 18.8 cm, while the long-term decline in primary consolidation is only 14.4 cm, so it meets the requirements of <15 cm.
5. Although the modeling of rib as Shell with concrete thickening on columns and encounters between ribs was decreased by 19 mm, but in the combined system of KSLL Foundation with 1100 m deep pile, the decreasing value was only 7.92 cm, so that the foundation structure of KSLL combination with Piling is very stable.

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